Recent Capabilities of CMS-Wave: A Coastal Wave Model for Inlets and Navigation Projects

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ABSTRACT

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The Coastal Inlets Research Program (CIRP) of the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) has developed a nearshore spectral wave transformation numerical model to address needs of the U.S. Army Corps of Engineers (USACE) navigation projects. The model is called CMS-Wave and is part of Coastal Modeling System (CMS) for wave estimates in the vicinity of coastal and estuarine navigation channels. It can simulate important wave processes at coastal inlets including wave diffraction, refraction, reflection, wave breaking and dissipation mechanisms, wave-current interaction, and wave generation and growth. This paper describes recent improvements in CMS-Wave that include semi-empirical estimates of wave run-up and overtopping, nonlinear wave-wave interactions, and wave dissipation over muddy bottoms. CMS-Wave may be used with nested grids and variable rectangular cells in a rapid mode to assimilate full-plane wave generation for circulation and sediment transport models. A brief description of these recent capabilities is provided.

ADDITIONAL INDEX WORDS: *Navigation, coastal inlets, numerical wave modeling, nearshore wave processes.*

INTRODUCTION

Improved and practical wave models are needed for USACE navigation mission dealing with wave transformation in the vicinity of coastal inlets, estuaries and river mouths, harbors, and navigation channels. Wave interaction with coastal jetties, breakwaters and revetments requires modeling wave reflection, diffraction, transmission and overtopping for accurate wave estimates of channel infilling and potential breaching or flanking of structures. Calculations are required for wave propagation across channels as well as over or through structures, where combined wave diffraction and transmission are present simultaneously. Wave run-up and overtopping of structures are frequently needed. To address these needs, a spectral wave transformation model called CMS-Wave has been developed to support the operation and maintenance of coastal inlet and navigation projects, also the risk and reliability assessment of shipping in inlets and harbors. It is a steady-state half-plane wave transformation model, where wave energy propagation is independent of time and calculated from seaward towards the coastal boundary. Wave reflection off coastal structures and beaches is included in CMS-Wave. The workings and performance of model have been described by Lin *et al.* (2006 and 2008), Seabergh *et al.* (2008), and Demirbilek *et al.* (2009).

The focus of this paper is to provide a brief description of some recent capabilities added to the model.

CMS-Wave can represent important coastal wave processes including diffraction, refraction, reflection, wave breaking and dissipation mechanisms, wave-current interaction, and wave generation and growth. The wave diffraction is calculated in the wave-action balance equation by an additional term. Recent features incorporated for grid nesting, variable rectangle cells, wave run-up and overtopping, and assimilation of full-plane wave generation are to support circulation and sediment transport studies. Details of the theory and numerical implementation are presented in the CMS-Wave technical report (Lin *et al.*, 2008), including a number of examples of practical applications. In this paper, we provide validation for wave run-up, transmission and overtopping of structures with laboratory data, wave dissipation over muddy beds and nonlinear wave-wave interaction with the field and experiment data.

WAVE-ACTION BALANCE EQUATION

CMS-Wave calculates the spectral wave transformation based on the wave-action balance equation (Lin *et al.*, 2008):

$$\frac{\partial C_x N}{\partial x} + \frac{\partial C_y N}{\partial y} + \frac{\partial C_\theta N}{\partial \theta} =$$
$$\frac{\kappa}{2\sigma}\left[\left(CC_g \cos^2 \theta N_y\right)_y - \frac{CC_g}{2}\cos^2 \theta N_{yy}\right] + S_{in} + S_{dp} + S_{nl}$$

$$(1)$$

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14. ABSTRACT

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where $N = E / \sigma$ is the frequency and direction dependent wave-action density, defined as the wave energy-density $E = E\,(x, y, \sigma, \theta)$ divided by the intrinsic frequency σ. N_y and N_{yy} denote the first and second derivatives with respect to y; x and y are the horizontal coordinates; θ is the wave direction measured counterclockwise from the x-axis; C and C_g are wave celerity and group velocity; C_x, C_y, and C_θ are the characteristic velocity with respect to $x, y,$ and θ, respectively. κ is an empirical parameter representing the intensity of wave diffraction effect. The right-hand side terms respectively are: S_{in} is the source (*e.g.*, wind input), S_{dp} is the sink (*e.g.*, bottom friction, wave breaking, whitecapping, *etc.*), and S_{nl} is the nonlinear wave-wave interaction.

The first term on the right side of Equation 1 is the wave diffraction term formulated from a parabolic approximation wave theory (Mase, 2001). In applications, the diffraction intensity parameter κ (≥ 0) needs to be calibrated and optimized for featured structures. The model omits the diffraction effect for $\kappa = 0$ and calculates the diffraction for $\kappa > 0$. In practice, the value of κ may range from 0 (no diffraction) to 4 (strong diffraction) for calculating diffraction effects. A constant value of $\kappa = 2.5$ has been used by Mase *et al.* (2001, 2005a, 2005b) to simulate wave diffraction for narrow and wide gap breakwater applications. Lin *et al.* (2008) and Demirbilek *et al.* (2009) demonstrate that value of $\kappa = 4$ is appropriate for semi-inifinite long breakwaters and also in narrow gaps (inlets) with openings equal or less than one wavelength. For wider gaps with the opening greater than one wavelength, $\kappa = 3$ is recommended. The exact value of κ in an application is dependent on the structure's geometry, local bathymetry and incident wave conditions, and may need to be fine-tuned with data. The default value of $\kappa = 4$ is used in the model, corresponding to strong diffraction. Implementation of wave diffraction is approximate, and phase-resolving wave models (Holthuijsen *et al.*, 2004) may be used to verify estimate of waves near structures, inlets and harbors.

Figures 1 and 2 show an example of the CMS-Wave computational domain and calculated wave results at four different locations (HMB1 to HMB4), respectively, at Half-Moon Bay in Grays Harbor entrance, Washington, USA. Osborne and Davies (2004) described the field data collection in Grays Harbor. Numerical simulation is conducted for December 10 through 31, 2003. The input wind and wave data were obtained from the National Data Buoy Center (NDBC - http://www.ndbc.noaa.gov) Station 46029 and Coastal Data Information Program (http://cdip.ucsd.edu) Buoy 036 (NDBC 46211), respectively. CMS-Wave was run in the coupled mode with a CMS-Flow model (Buttolph *et al.*, 2006; http://cirp.usace.army.mil/products) that provided calculated water level and current input fields. The effect of current on waves (H_s shown in Figure 2 is the significant height defined as the mean of the highest 1/3 wave height) is pronounced at gauges HMB1 and HMB2 located closer to the navigation channel in relatively deep water. Comparison of simulation results with the field data shows that shallow water effects on wave diffraction, refraction, and breaking are evident at HMB3 and HMB4.
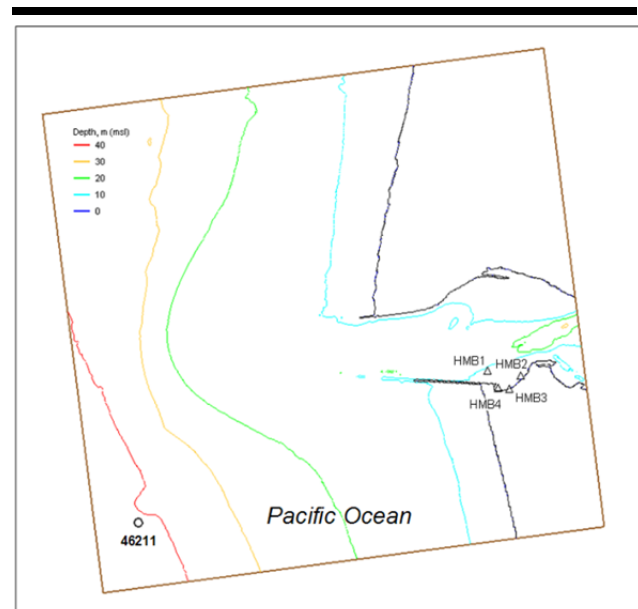

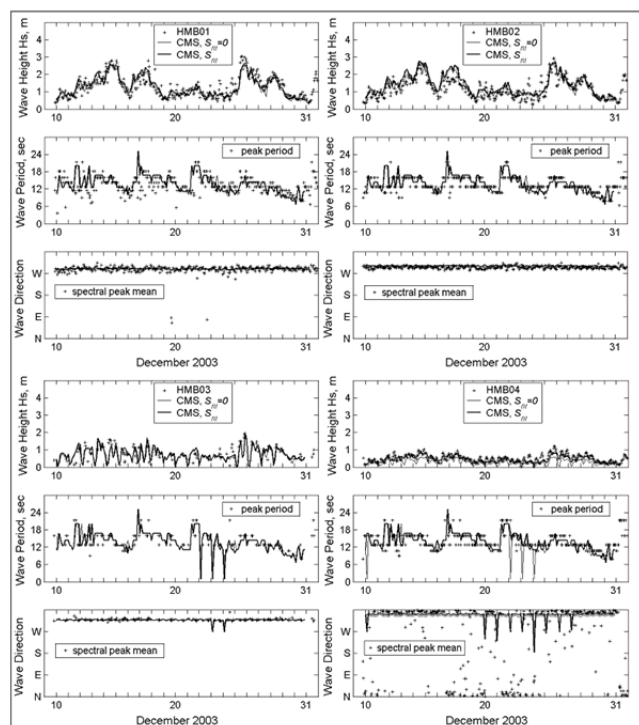Figure 1. CMS-Wave grid and data-collection stations at Grays Harbor, WA. USA.

Figure 2. Measured and calculated waves at HMB1 to 4, 10-31 December 2003.

WAVE RUN-UP

The wave run-up consists of two components: (a) a rise of the mean water level resulting from wave breaking at the shore, known as the wave setup η, and (b) swash oscillation of the incident waves. The wave setup in CMS-Wave is computed from the horizontal momentum equations by neglecting current, surface wind drag and bottom stresses as

$$\frac{\partial \eta}{\partial x} = \frac{1}{\rho g h} \left(\frac{\partial S_{xx}}{\partial x} + \frac{\partial S_{xy}}{\partial y} \right) \qquad (2)$$

$$\frac{\partial \eta}{\partial y} = \frac{1}{\rho g h} \left(\frac{\partial S_{xy}}{\partial x} + \frac{\partial S_{yy}}{\partial y} \right) \qquad (3)$$

where ρ, g, h are the water density, gravitational acceleration and water depth, respectively. Three radiation stress components S_{xx}, S_{xy}, S_{yy} are calculated using the linear wave theory (Dean and Dalrymple, 1984) as

$$S_{xx} = \iint E \left[n(\cos^2\theta + 1) - \frac{1}{2} \right] d\theta \, d\sigma \qquad (4)$$

$$S_{yy} = \iint E \left[n(\sin^2\theta + 1) - \frac{1}{2} \right] d\theta \, d\sigma \qquad (5)$$

$$S_{xy} = \iint \frac{E}{2} n \sin 2\theta \, d\theta \, d\sigma \qquad (6)$$

where $n = \frac{1}{2} + \frac{kh}{\sinh 2kh}$ and k is the wave number. The swash oscillation of waves on the beach face is a random process. In engineering applications, the 2-percent runup defined as exceedance of the vertical level is commonly used and denoted as $R_{2\%}$ (or $R2$). Komar (1998) gives an estimate of $R_{2\%}$ on beaches, seawalls and jetties as

$$R_{2\%} = 2|\eta| \qquad (7)$$

The $R_{2\%}$ is calculated using Equation 7 at the land-water interface and averaged with the local depth to determine if water can flood the dry cells. If the wave run-up level is higher than elevation of adjacent dry land cells, these cells are flooded to simulate the overtopping and overwash at these cells.

Wave run-up calculation in CMS-Wave is verified with data collected from two laboratory studies of random wave up-rush on plane smooth slopes (Ahrens and Titus, 1981; Mase and Iwagaki, 1984). There are together a total of 395 run-up cases from these two studies covering a broad range of plane slope (1:1 to 1:30) and incident spectrum. Ahrens and Titus (1981) characterized the incident wave by the spectral peak frequency and a wave group parameter (Goda, 1970) defined as

$$Q_p = \frac{2\int \sigma E^2(\sigma) d\sigma}{[\int E(\sigma) d\sigma]^2},$$

where
$$E(\sigma) = \int E(\sigma, \theta) d\theta \qquad (8)$$

while Mase and Iwagaki (1984) specified the incident wave by JONSWAP spectrum (Hasselmann *et al.*, 1973) as follows:

$$E_J(\sigma) = $$
$$\frac{\alpha g^2}{\sigma^5} \exp\left\{ -1.25(\frac{\sigma}{\sigma_m})^4 \right\} \exp\left\{ \ln \gamma \exp[-0.5(\frac{\sigma - \sigma_m}{\beta \sigma_m})^2] \right\} \qquad (9)$$

where α, β are scale and shape parameters, and σ_m, γ are spectral peak frequency and peak-enhancement factor, respectively. Incident waves in Ahrens and Titus are converted to a JONSWAP spectrum as input to CMS-Wave using the relationship of $\gamma = 2(Q_p - 3) + 3$ derived from Equations 8 and 9.

Figure 3 shows the calculated and measured $R_{2\%}$. Overall, calculated run-up estimates agree with data; the correlation coefficient for all cases is 0.83 (Lin *et al.*, 2008). For steeper slopes (1:1 to 2:3), the model tends to overpredict $R_{2\%}$ suggesting Equation 7 is more accurate for mild slopes.
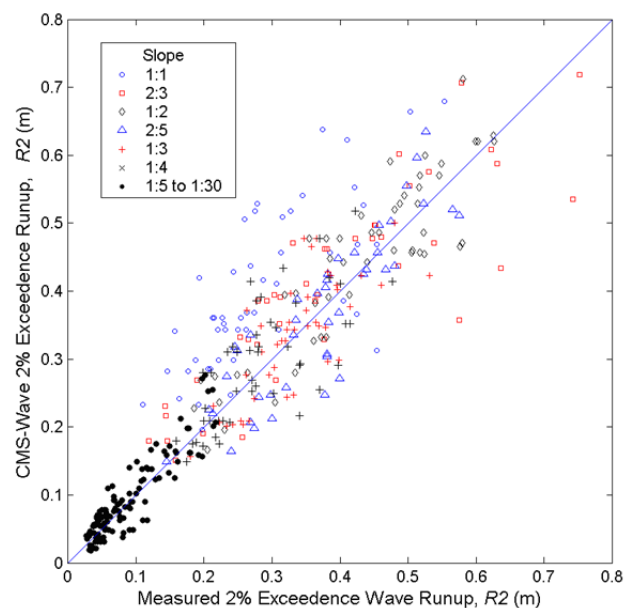

Figure 3. Measured and calculated 2% exceedence wave runup.

WAVE TRANSMISSION AND OVERTOPPING OF STRUCTURES

Wave transmission over low-crested or submerged impermeable structures is exhibited by fall of the overtopping water mass. The ratio of structure crest elevation to incident wave height is a key parameter governing the wave transmission. CMS-Wave calculates the transmission coefficient K_t, defined as the transmitted wave height divided by the incident wave height, based on the simple expression (Goda, 2000):

$$K_t = 0.3 \ (1.5 - \frac{h_c}{H_i}), \ \text{ for } \ 0 \le \frac{h_c}{H_i} \le 1.25 \qquad (10)$$

where h_c is the crest elevation of the breakwater above the still-water level (h_c is negative for a submerged breakwater), and H_i is the incident wave height. For a composite breakwater, protected by a mound of armor units at its front, K_t is calculated (Goda, 2000) as

$$K_t = 0.3 \ (1.1 - \frac{h_c}{H_i}), \ \text{ for } \ 0 \le \frac{h_c}{H_i} \le 0.75 \qquad (11)$$

For permeable rubble-mound breakwaters, the transmission is calculated using d'Angremond *et al.* (1996) formula as

$$K_t = 0.64 \left[1 - \exp\left(-\frac{\xi}{2}\right) \right] (\frac{B}{H_i})^{-0.31} - 0.4 \frac{h_c}{H_i}, \text{for } B < 10 \ H_i \qquad (12)$$

where B is the crest width and ξ is the Iribarren parameter defined as the fore-slope of the breakwater divided by the square-root of deepwater incident wave steepness. In practice, Equations 10 to 12 are applicable to both monochromatic and random waves. Figure 4 shows the calculated transmission coefficients and data curves compiled by Goda (2000) for a vertical breakwater. In these simulations, CMS-Wave was forced by a monochromatic wave of 1 m and 6 sec for a vertical breakwater with h = 10 m, d = 5 m, and B = 20 m. Table 1 shows comparison of calculated transmission coefficients and laboratory data compiled by Goda (2000), and Equations 10 and 11 for monochromatic incident waves.
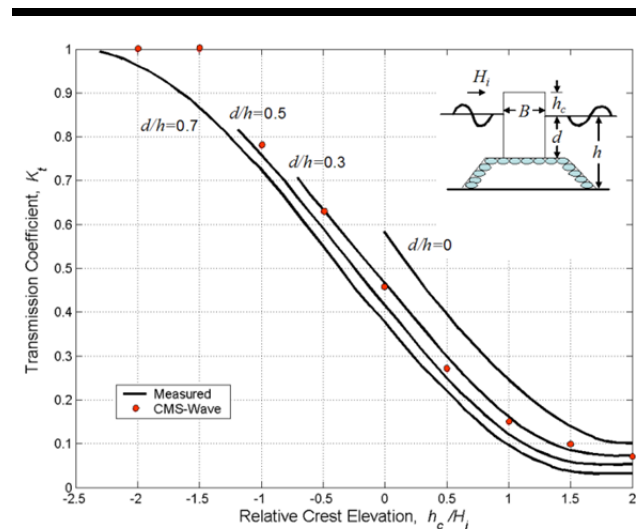

Figure 4. Measured and calculated transmission coefficients for a vertical breakwater.

Wave overtopping rate is calculated using two different methods. In a coupled CMS-Wave and CMS-Flow simulation (*e.g.*, flow model gets radiation stresses from wave model and provides calculated current and water levels to wave model), the overtopping rate is the flow rate calculated by CMS-Flow. Without coupling, CMS-Wave calculates the average overtopping rate using the empirical formula of Hughes (2008) given by

$$q = \sqrt{gH_i^3} \left[0.0336 + 0.53 \left(\frac{h_c}{H_i} \right)^{1.58} \right] \qquad (13)$$

Table 2 provides comparison of the calculated average overtopping rates obtained with decoupled and coupled CMS runs performed for laboratory measurements of Hughes (2008) for four incident irregular waves and two steady storm surges overtopping a levee. In the simulation of higher surge (water level = 1.3 m) and larger waves (incident wave height = 2.3 m), the calculated overtopping rates from the CMS-Wave alone agree better with data than results from the coupled run. This suggests that empirical formulas used in CMS-Wave for overtopping, runup and transmission need further verification with data to determine appropriateness of each formula.

Table 1. *Comparison of transmission coefficients, K_t*

h_c (m)	Vertical Breakwater			Composite Breakwater	
	Data (Goda, 2000)	Eq. 10	CMS-Wave	Eq. 11	CMS-Wave
-2			1.02		1.02
-1.5			1.03		1.03
-1	0.76		0.78		0.78
-0.5	0.60		0.63		0.63
0	0.42	0.45	0.46	0.33	0.34
0.5	0.26	0.30	0.27	0.18	0.18
1	0.13	0.15	0.15	0.03	0.04
1.5	0.06		0.10		0.024
2	0.05		0.07		0.018

Table 2. *Comparison of average overtopping rate (m^2/sec)*

Exp No.	Surge Level (m)	Wave Height (m)	Wave Period (sec)	Measured (Hughes, 2008)	Calculated	
					CMS	CMS-Wave
R128	0.29	-	-	0.27	0.28	-
		0.82	6.1	0.38	0.38	0.39
R109	0.29	-	-	0.26	0.28	-
		2.48	13.7	0.70	0.85	0.92
R121	1.3	-	-	2.55	2.57	-
		2.3	6.1	2.67	2.93	2.76
R127	1.3	-	-	2.54	2.57	-
		2.3	14.4	2.84	2.98	2.81

WAVE-WAVE INTERACTIONS

The exact solution of wave-wave interaction requires solving computationally expensive six-dimensional integral (Resio and Tracy, 1982). The wave-wave interactions in CMS-Wave are

calculated using a modified method of Jenkins and Phillips (2001). They proposed a simple formula to represent wave-wave interactions as a second-order diffusion operator of the isotropic form that conserves the wave action. CMS-Wave implements this formula directly in the wave-action balance equation such that no additional integration is required to calculate wave-wave interactions. By keep only the first and second-order terms in the Jenkins-Phillips formulation, wave-wave interactions in finite water depth can be expressed as

$$S_{nl} = a\frac{\partial F}{\partial \sigma} + b\frac{\partial^2 F}{\partial \theta^2} \qquad (14)$$

where $a = \frac{1}{2n^2}[1+(2n-1)^2\cosh 2kh]-1$ is a function of kh, $b = \frac{a}{n\sigma}$,

and

$$F = k^3\sigma^5\frac{n^4}{(2\pi)^2 g}\left[(\frac{\sigma_m}{\sigma})^4 E\right]^3 \qquad (15)$$

The variation of a and $b\sigma$ as functions of kh is shown in Figure 5, indicating that wave-wave interactions are more significant in the intermediate water depths and diminish in the shallow water. This is because wave-wave interactions initially take place in the deepwater, continue to evolve in the intermediate depth, and gradually diminish in shallow water. The effect of interactions on wave evolution is greater over long fetches in a large ocean domain and less in a local coastal region. Figure 6 shows the comparison of directionally integrated S_{nl} from Equation 14 and exact computations in the examples of Hasselmann *et al.* (1985). These calculations are for the JONSWAP spectra with $\gamma = 2$ and $\gamma = 5$. Figure 7 shows the calculated nonlinear wave energy transfer rate S_{nl} in the frequency and direction domain for a JONSWAP spectrum with $\gamma = 5$. These calculated results are consistent with the observed and theoretical results that the nonlinear wave-wave interactions cause wave energy to transfer from high to low frequencies (downshifting).

Figure 8 shows the calculated wave height fields with and without nonlinear wave-wave interactions at the Louisiana coast in the north central Gulf of Mexico. The input wind and incident wave data are supplied by NDBC Buoy 42041 located approximately in the middle of offshore boundary. Figure 9 shows the corresponding wave period fields with and without the nonlinear wave energy transfer. The simulation is for the growth of an incident deepwater southeast wave of 0.3 m and 4 sec under a moderate 15-kt wind from southeast. With the nonlinear wave-wave interactions, the calculated wave height along the coast is about 10 percent higher than without the nonlinear energy transfer effect. The nonlinear wave-wave interactions also increase wave periods over a larger coverage area, which is a consequence of wave energy transfer from higher to lower frequencies. Wind input was triggered in these simulations. It should be noted that wind input can reveal better wave-wave interaction result because more wave energy from wind input can transfer from high to low frequencies. The wave-
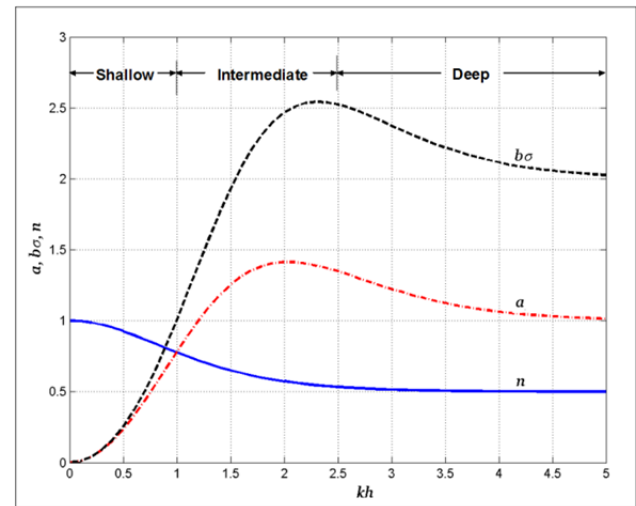

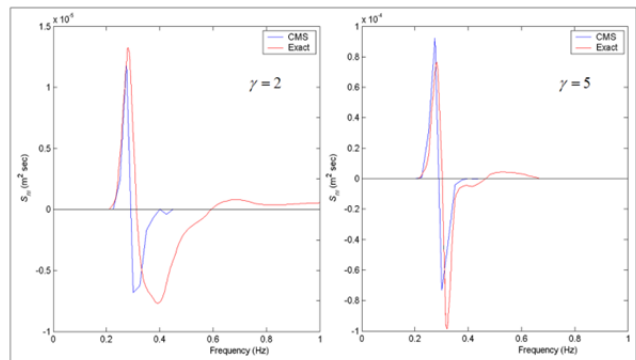Figure 5. Nonlinear wave-wave interaction coefficients a and $b\sigma$ as functions of kh.

Figure 6. Comparison of directionally integrated S_{nl} for JONSWAP spectrum with γ =2 and 5.

wave interaction can be rather small and insignificant in magnitude without the wind input.

The wave transformations near inlet jetties and in the wave diffraction zone can be improved with the nonlinear wave-wave interactions. Figure 2 shows the comparison of calculated waves with and without the nonlinear energy transfer in Grays Harbor. The calculated wave height with the nonlinear energy transfer is about 5 percent higher than without the nonlinear energy transfer. This change is small and in practical applications can be neglected. More importantly, we note that the proposed new formulation that extends the Jenkins-Phillips original formulation from deepwater to finite depth should be
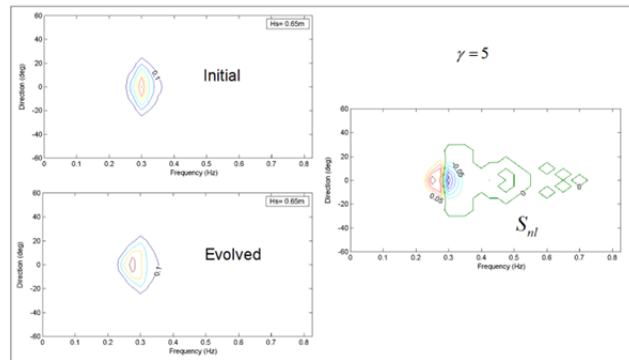
Figure 7. Calculated non-linear energy transfer for a directional JONSWAP spectrum with γ =5.
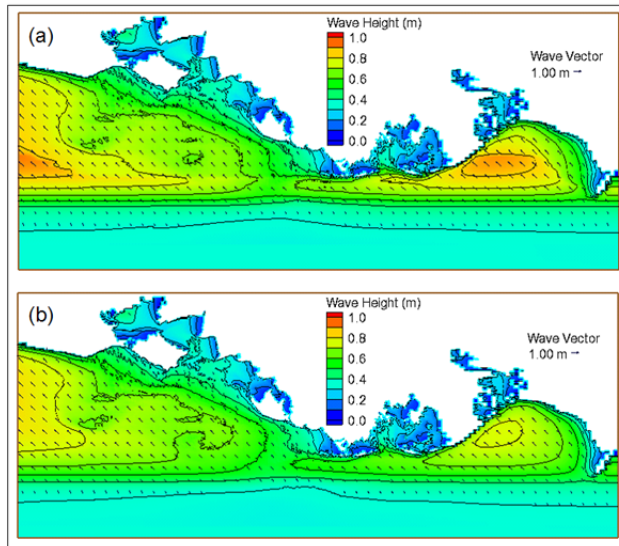

Figure 8. Calculated wave height field (a) with and (b) without non-linear wave energy transfer.

extensively tested in different depths with different wave and wind conditions. The robustness and consistency of our proposed formula must be validated with laboratory and field data and confirmed against other formulations.

WAVE DISSIPATION OVER MUDDY BOTTOMS

The wave dissipation over muddy beds in CMS-Wave is calculated based on the assumption that the turbulent eddy viscosity is several orders of magnitude greater than the kinematic viscosity of sea water. By neglecting the kinematic
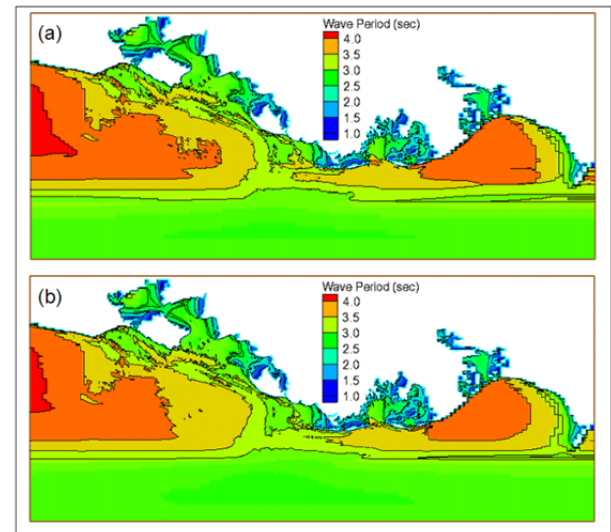

Figure 9. Calculated wave period field (a) with and (b) without non-linear wave energy transfer.

viscous effect, the wave dissipation over a muddy bed can be expressed as (Lamb, 1932)

$$S_{dp} = -4(\nu_k + \nu_t)k^2 E \qquad (16)$$

where ν_k is the kinematic viscosity and ν_t is the turbulent eddy viscosity. In the present study, ν_t is formulated by a maximum viscosity ν_{tb} representing the wave breaking condition times the ratio of wave height over depth as $\nu_t = \nu_{tb}\dfrac{H_s}{h}$. The value of ν_{tb} is set to 0.04 m^2/sec for the muddy bed and 0.01 m^2/sec for the primary sand area based on model comparison with data. As an example, wave fields affected by muddy beds for the Louisiana coast are calculated and shown in Figures 8 and 9. In Figure 10, CSI3 is the location of muddy bed, and CSI5 is for the primary sandy bed. These wave gauges were deployed by the Coastal Studies Institute (CSI) at Louisiana State University (Sheremet and Stone, 2003; Sheremet et al., 2005). Figure 11 shows the comparison of calculated and measured wave heights at gauges CSI3 and CSI5. The calculated wave heights at the muddy bed CSI3 using Equation 16 agree better with measurements as compared to those without the muddy bed effect included. For the more sandy location CSI5 away from the muddy area, the calculated wave heights are less affected by inclusion of the muddy bed effect .The present treatment of muddy bed does address the potential affect of mud on dynamics of waves as reported by other researchers (Sheremet, 20005; Kaihatu, 2008). The goal in the present study is to provide approximate estimate of mud on spectral wave dissipation, without using any complicated approaches.
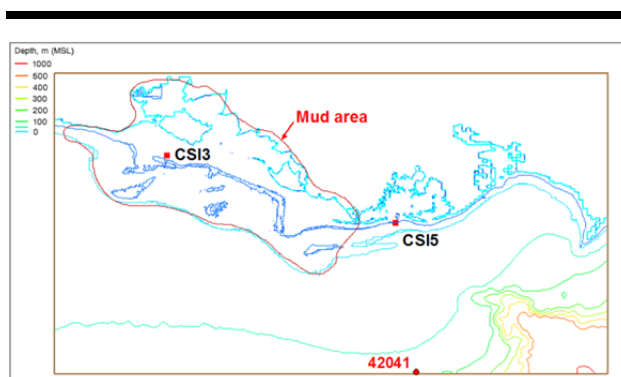
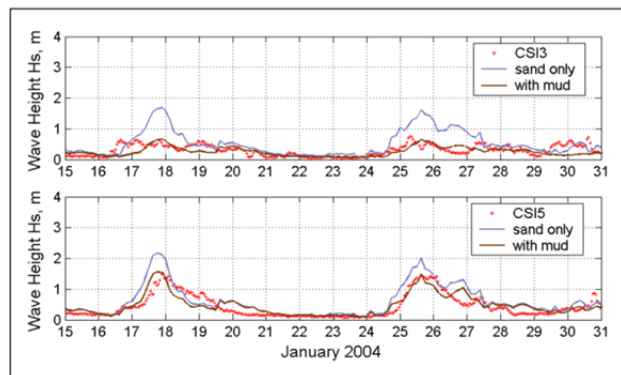Figure 10. Location of muddy bed, NDBC 42041, and CSI wave gauges.

Figure 11. Measured (+) and Calculated (---) waves on muddy bed along Louisiana coast.

CONCLUSIONS

This paper introduces some recent capabilities that have been added to CMS-Wave model. Due to space limitation, only a brief description of the added features is provided and details of these will be reported soon in technical reports and notes. Examples provided for the new capabilities are simply comparisons to analytical solutions, other models and available data. The comparisons include wave run-up, wave transmission over and through structures, wave-wave interactions, and wave energy dissipation on muddy beds. These new features implemented in CMS-Wave may not be essential, but often are needed for wave estimates in coastal engineering applications near structures and in navigation channels, where wave-current and wave-structure interactions can become important and significant. Additional research is in progress to further evaluate the appropriateness of the proposed capabilities, which require extensive validation and verification before these new features can be confidently used in the engineering practice. Once completed, a comprehensive mathematical description of these new added features will be provided with validation examples.

Other features can also be important concerning wave asymmetry in shallow water, infra-gravity waves for seiching in harbors, and surface roughness of breakwaters. Future companion papers in this series will address a more complete description of these nearshore wave processes.

ACKNOWLEDGMENTS

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